

JACK-UP RIG FOUNDATION DESIGN APPLYING 3D FE STRUCTURE - SOIL - INTERACTION MODELLING

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SUMMARY: The foundation engineering assessment for the world's largest jack-up rig installed offshore Norway is carried out. Based on the site survey and soil investigation data the soil conditions vary across the site and consist of bedrock overlaid by very soft silt / clay and a shallow layer of seabed sand. From the conventional and finite element analyses an engineering solution comprising construction of sand banks was proposed. The final geometry of the banks is determined based on the three – dimensional (3D) finite element (FE) integrated jack-up structure – skirted spudcan – soil – interaction modellings documenting that during the rig installation / preloading and storm loading the structural forces fall within the accepted limits. The rig was successfully installed and upgraded verifying the engineering predictions.

INTRODUCTION

In the design and installation of different offshore structures an important aspect is the investigation of the seabed conditions considering features such as slopes, depressions, boulders, seabed inclination etc. Particularly for a jack-up drilling rig, which is supported by three independent legs at an approximate distance of (50 – 60) m from each other, this is a very important issue.

In addition to seabed features the variation of the soil conditions in depth and horizontal directions, at the three footing locations plays also a decisive role in the structure and foundation solution.

A geotechnical engineering assessment for the world's largest jack-up rig is given in

this paper. The rig was temporarily installed next to a quay of a Norwegian yard in order to be upgraded for further production work in the North Sea.

The preliminary site survey indicated that the seabed in the considered area of location was rock outcrop, undulating across the site creating slopes and depressions.

As the current rig's footings are skirted spudcans with areas exceeding the plan dimensions of the seabed features and the skirts cannot penetrate through the rock, seabed modification was necessary. The idea of creating flat areas at the footing's locations, through construction of shallow gravel / sand banks to a height slightly larger than the skirt heights, was initially proposed.

After the preliminary site survey a detailed geotechnical investigation was carried out to verify the soil conditions. From the investigation a sediment layer of varying thickness overlying the undulating bedrock was identified.

Preliminary conventional and numerical skirted spudcan penetration analyses, with originally low and lately rather increased elevations of the sand banks, indicated non-uniform penetration, sliding and rotation of the free skirted spudcans under preloading conditions. For a more realistic evaluation, jack-up structure - foundation interaction effects were accounted for in the analyses.

The full 3D FE jack-up structure – skirted spudcans – sand bank – soil - interaction models were applied for varying heights of the sand banks, investigating the non-uniform footings penetrations, rotations and sliding and how they effect the structure. After an extensive amount of analyses, the final heights of the sand banks are proposed.

JACK-UP STRUCTURE – SKIRTED SPUDCAN - SOIL SYSTEM

The current jack-up drilling rig, the world's largest, is type independent leg cantilever and is shown in Fig. 1 in comparison to other world known structures. It operates in water depths up to 150 m and it has leg lengths of about 205 m.

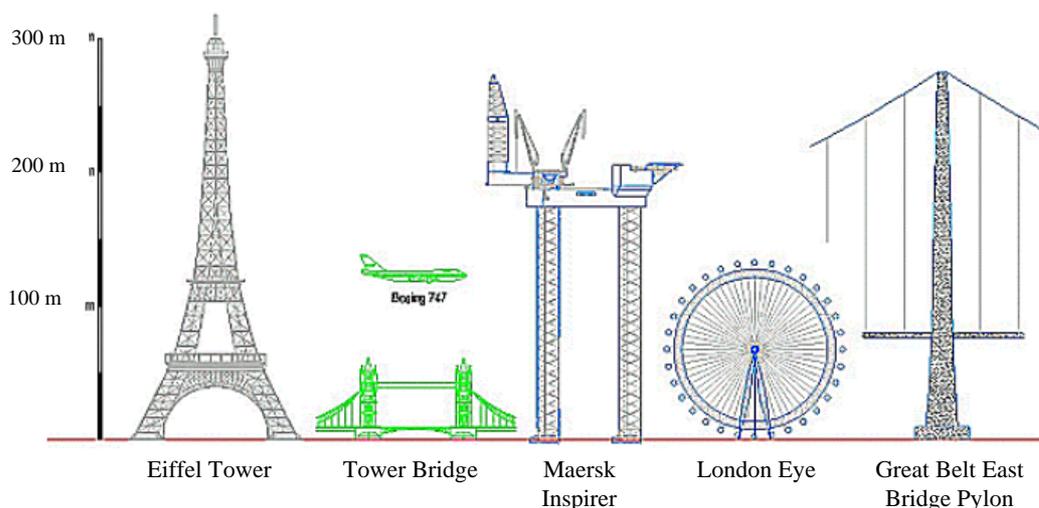


Fig. 1. The jack-up rig in comparison to other structures

Structure elements and stiffness

In the 3D FE jack-up - foundation - interaction model only the main structure elements such

as the three legs and the hull were included. The legs were simplified to 3D beam elements, and the hull to plate or floor elements with the equivalent thickness. The rig designer provided the geometry data for the legs and the hull.

Skirted spudcan geometry

The considered jack-up footings have a diameter $D = 22$ m and are fitted with outer and internal skirts, which divide the spudcan into 6 compartments. Fig. 2 shows photo views of the spudcan inside and a 3D model.



Fig. 2. Jack-up skirted spudcan

The vertical geometry of the spudcan structure is mainly given by: Distance from spudcan base to tip of outer skirts 2.3 m; Distance from spudcan base to tip of internal skirts 1.1 m; The spudcan itself is almost a flat rigid plate. The transverse stiffnesses of the skirts are derived from the structural FE model of the spudcan. These thicknesses are applied in the 3D FE analyses employing wall structural elements, respectively.

Interpretation of the soil conditions

Except the preliminary site survey, to identify the seabed and the soil conditions at the considered spudcan locations a new site survey, seismic, (sparker and pinger) and bathymetry was carried out. Sediments of varying thickness overlaying the bedrock were interpreted. The largest sediment thicknesses were seen at the largest water depth. Gravity vibrocore samples taken from seabed could not reach the bedrock and showed mostly sediments of clayey, gravely sand. At the shallow water depths the bedrock outcrops the seabed.

After the interpretation of the seismic survey geotechnical investigation including 5 piezo-cone penetration tests (PCPTs) and one vibrocore for each spudcan location were carried out. Discrepancies were though recognized at some PCPT locations. Considering the limitations of the sparker survey, a pinger survey was carried out. With a less penetrating, but a smaller opening angle seismic source, the pinger survey was applied to better identify the slope of the bedrock and supplement the previous investigations in the area. Based on the pinger data combined with the existing and new soil information, a re-interpreted model of the sediment and bedrock surface was produced.

Several originally proposed locations were studied finally focusing on one of them. At each leg position four vertical cross sections such as north - south, west - east, northwest - southeast, northeast - southwest were interpreted showing the seabed depth and the depth of the top bedrock from the centre of the spudcans out to a distance of 50 m.

To estimate the variation in thickness of the soil layers between the bedrock and the seabed and the soil parameters applicable to the design of the foundations at the final location a new geotechnical investigation consisting of 5 boreholes, about 70 PCPTs and laboratory tests, classification and simple and advanced strength tests were carried out.

From the engineering interpretation of the gathered soil data it was assessed that the soil conditions consist of overall quaternary marine sediments, mainly deposits consisting of a seabed layer of sand overlying clayey, sandy silt with variable thickness (0 - 9) m overlying crystalline bedrock.

Water depth variation

The water depth at the centre of the three chosen spudcan locations are as seen from Fig. 3, approximately -23 m at spudcan S1, -19 m at S2 and -26.5 m at S3. So the location of S2 is shallowest, becoming deeper at S1 and even deeper at S3.

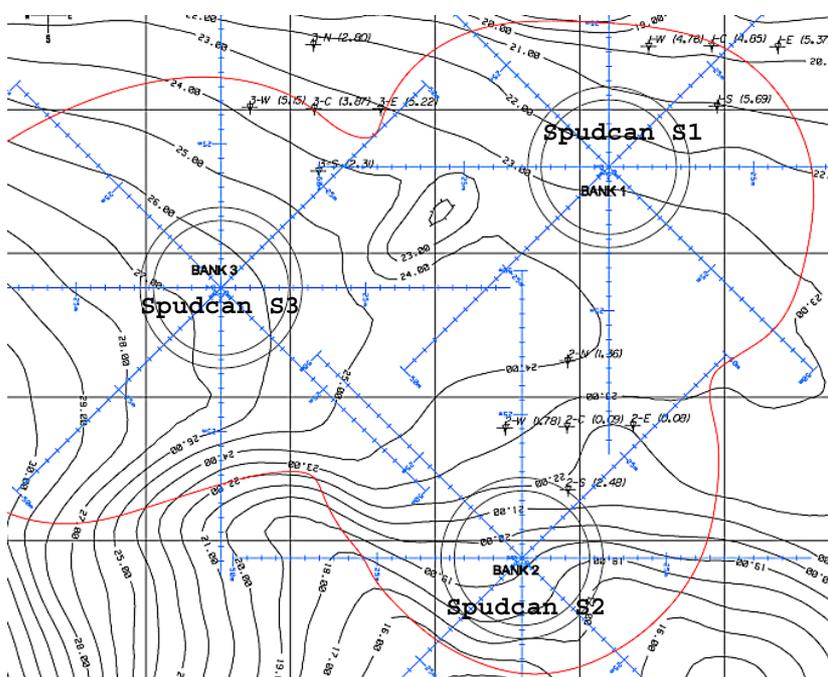


Fig. 3. Bathymetry at the considered rig location

Design soil profiles and parameters

On the basis of the seismic surveys, PCPTs / boreholes and laboratory test results (classification and triaxial, unconsolidated undrained (UU) and consolidated isotropic drained (CID)) the soil profiles and soil parameters applicable to the engineering assessment of the jack-up at the considered location are derived.

A good correlation with the PCPT data was investigated for the depths where soil samples were taken and laboratory tests performed. However, at the depths where lower cone strength as shown in Fig. 4 and Fig. 5 were recorded no soil sample could be extracted and no correlation was available. Under these circumstances the cone factor $N = q_{net} / c_u = (15 - 20)$ was used. When applying such a correlation on the PCPT data undrained shear strength for the silt $c_u = (15 - 25)$ kPa were assessed.

Based on the test results and the engineering judgement initially $c_u = 25$ kPa for the silt and a friction angle $\phi = 35^\circ$ for the seabed sand were applied.

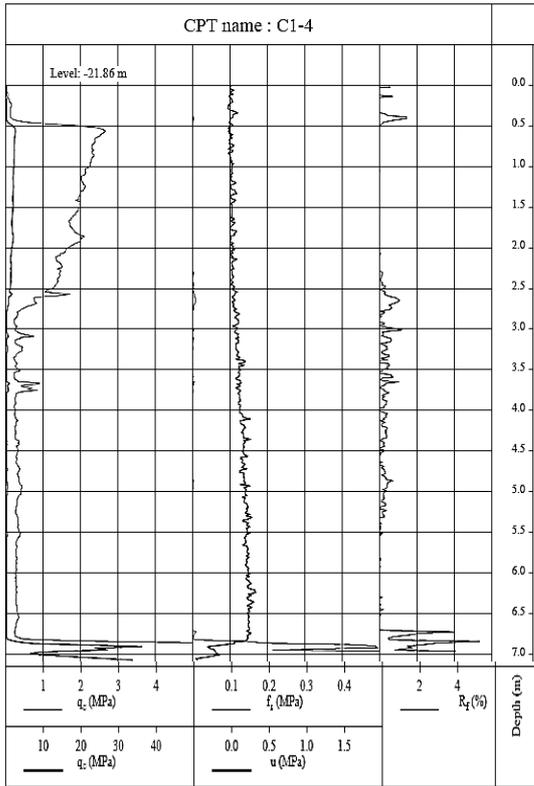


Fig. 4. PCPT profile at S1 location

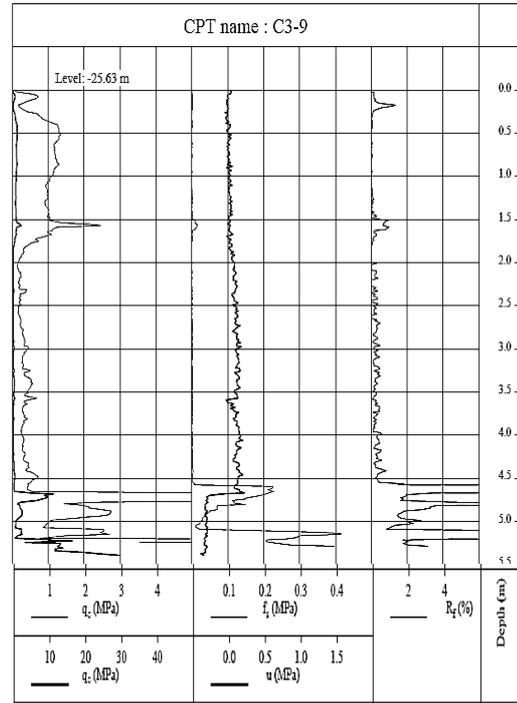


Fig. 5. PCPT profile at S3 location

The soil parameters for the bedrock are evaluated based on the engineering experience. An undrained shear strength $c_u = (1000 - 1500)$ kPa was assigned.

‘As it was’ soil profile at the three spudcan location can be given by Table 1. There are three soil layers throughout the site. However, the thickness and the elevations of the layers strongly vary not only between the spudcan locations but also within the spudcan areas.

Table 1. ‘As it was’ soil profile at the considered jack- up footings locations

Soil description	Depth of layer h (m)	Unit weight γ' (kN/m ³)	Angle of internal friction and dilation		Undrained shear strength c_u (kN/m ²)	Deformation Modulus E (kN / m ²)
			ϕ (°)	ψ (°)		
SAND, dense	Variable	10.0	35	5	-	35000
SILT / CLAY, very soft to soft	Variable	9.0	-	-	25 / 15	100 * c_u
BEDROCK	Variable	12.0	-	-	1500 / 1000	200 * c_u

The material of the sand bank constructed over the seabed at the spudcan locations is modelled applying a unit weight $\gamma' = 11$ kN/m³, a friction angle $\phi = (40 - 45)^\circ$ and a dilation angle $\psi = (10 - 15)^\circ$. The deformation parameters $E = 100000$ kPa was assigned.

The variation of the soil conditions with depth through the considered jack-up area is shown in the next sections showing different cross sections where the final sand banks

proposed for stabilizing the foundations are included.

CONVENTIONAL AND NUMERICAL ANALYSES

Skirted spudcan capacities / stabilities under preloading conditions are investigated applying different methods of foundation analyses consisting of conventional and 2D / 3D FE modelling. The common conventional skirted spudcan penetration analyses are not directly applicable in this case as the thickness of the soil layers vary within the spudcan area and a uniform soil profile can not be assumed.

The results from the 2D FE analyses of the free-skirted spudcans – soil interaction considering shallow sand banks, were found conservative as well. Therefore, 3D FE modelling was performed assigning boreholes at the location where soil profile changes.

The preliminary analyses applying $c_u = 25$ kPa for the silt layer showed large rotations and horizontal movement for the free skirted spudcans S1 and S3 where the thickness of the silt layer is considerable and variable. Spudcan S2 is found more stable as the silt layer is almost negligible and the height of the sand bank is conditioned from the skirt depth.

An attempt to include some structural stabilizing loads considering leg-spudcan rigidity while preloading the spudcans was made. The iterative procedure was found cumbersome and therefore cancelled.

The issue of structure – foundation - soil interaction was raised at this time. The idea of a full 3D model consisting of jack-up structure – skirted spudcans founded in the sand banks constructed over the underlying soil was explored. The height of the gravel banks were then to be determined based on the limited structural forces evaluated from the rig designer. The structure – foundation system was investigated under preloading and storm loading conditions.

Conventional skirted spudcan differential penetration

Before building the full 3D structure foundation model, which is complicated and requires time for calculation, an investigation on the effect of the sand bank heights on the skirted spudca bearing capacity and differential penetration at the locations S1 and S2 is carried out. The minimum height at S2 is already defined.

Two extreme soil profiles within the spudcan area are chosen, which are expected to give maximum and minimum penetrations. The differences in penetrations are interpreted as differential penetrations of the free-skirted spudcan. The method is not considered accurate for the final design but serves as preliminary evaluation.

To conventionally define footing penetration depth versus load, calculation of the static bearing capacity of the spudcan and the skirt elements at various depths, is carried out. When the spudcan reaches full base contact the assumption for an embedded footing is applied. This analytical calculation method is previously verified from the FE modelling of the skirted spudcan resting on different soil conditions.

The spudcan bearing capacity is calculated conventionally based on SNAME¹ and Hansen's theory² and a program developed from the experience with spudcan penetration predictions. The spudcan is simplified to a circular footing having a flat bottom. However, the effect of the actual spudcan shape is taken into account.

When frictional soil overlies firm to stiff clay, punching failure is investigated using a load spread factor 1:3, assumption applied in connection with the skirt effect. This is

verified from previous FE analysis of the spudcan penetration in sand-clay soil profiles. In addition, squeezing of the clay soil below the spudcan skirt is taken into account as well.

The penetration resistance of the skirt elements is calculated as the sum of the end resistance and skin resistance. The calculated bearing areas and the skin friction areas are applied, respectively. Danish Code of Practice for the pile theory is used.

To see the effect of the sand bank height in the differential penetration at spudcan S1 the elevation of the bank is moved up from -21 m to -19 m to -14 m. No punch through risk was expected for any of the scenarios. The results for sand bank at elevation -21 m and -19 m are given for illustration in Fig. 6. The maximum preload 145 MN / leg is calculated.

The conventional analyses for the soil profiles giving maximum penetrations are also compared with some FE axisymmetric analyses of the spudcan penetration as shown in Fig. 6. Such analyses are previously carried out by Kellezi & Stromann³, Kellezi et al.^{4,5} etc.

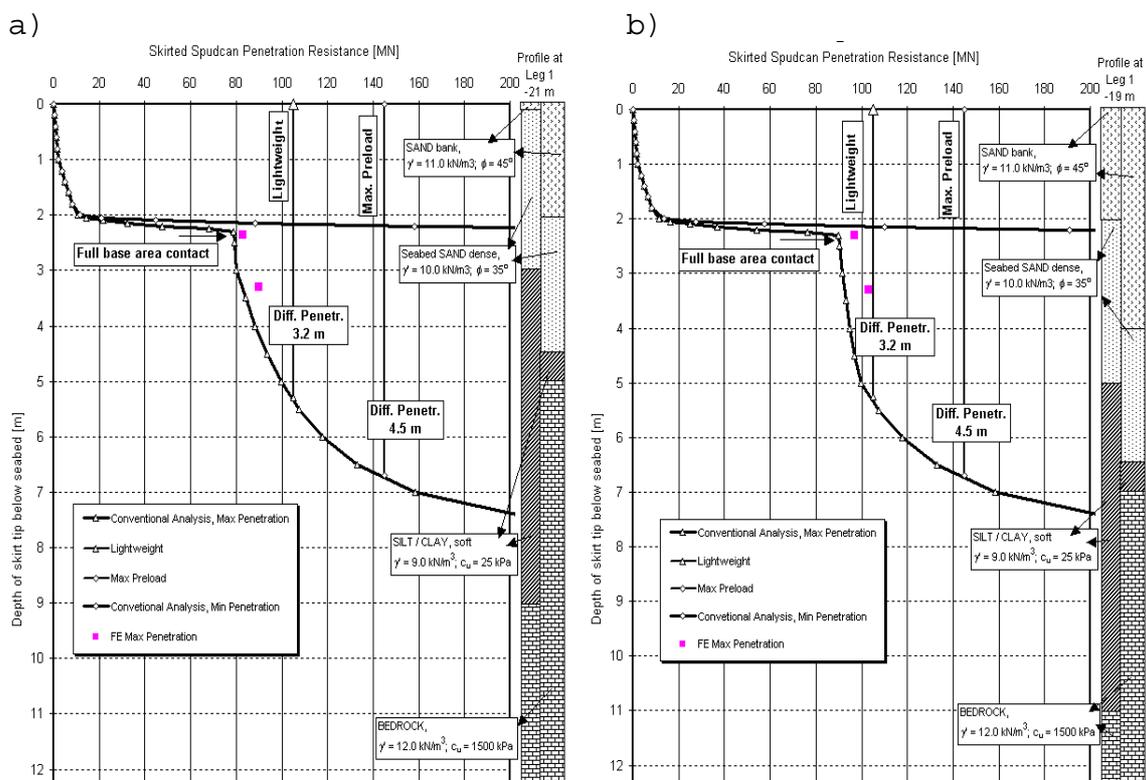


Fig. 6, Conventional skirted spudcan differential penetration analysis assuming extreme soil conditions at S1: a) Sand bank at -21 m; b) Sand bank at -19 m;

For spudcan S2 the height of the sand bank is equal to the length of the skirt / chord, plus some tolerance. The top of the bank was assumed at -13.5 m and small differential penetrations were predicted.

For spudcan S3 two extreme soil profiles are chosen as well expected to give maximum and minimum penetrations. The elevation of the gravel bank is moved up from -25 m to -23 m to -21 m to -18 m to -16 m. No punch through risk is expected.

To make the location applicable for the rig installation, high sand banks were necessary based on the conventional and FE axisymmetric results applying the soil mechanic principle of load spreading. Higher banks increase the bearing capacity of the silt layer as a result of increasing 'fictive' bearing area.

3D FE jack-up – skirted spudcan – sand banks - interaction modelling

The full 3D model is shown in Fig. 7 and 8. Different models with slightly different heights of the gravel banks were investigated.

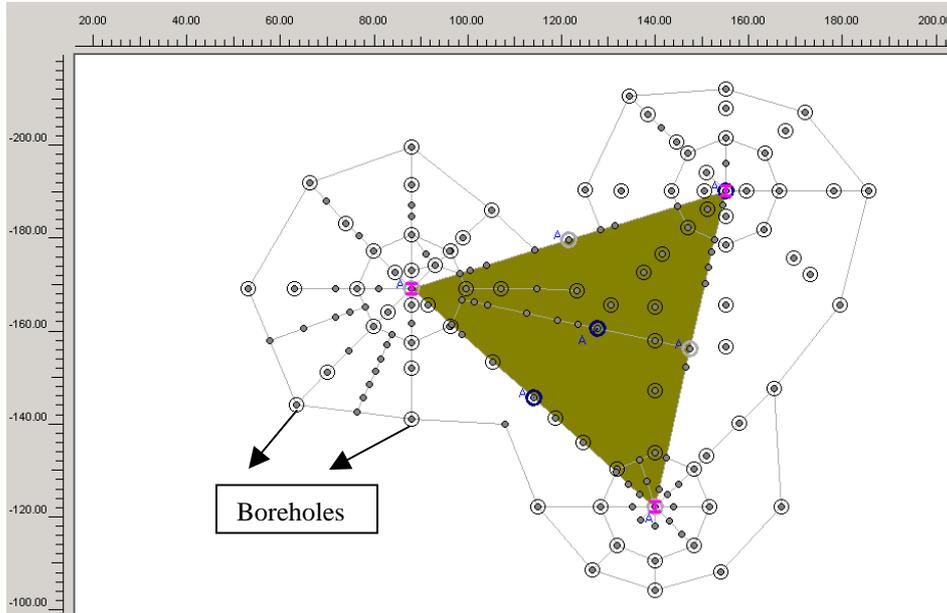


Fig. 7. 3D FE structure-foundation model, 2D build-up, horizontal plane at Hull Level +4.0 m

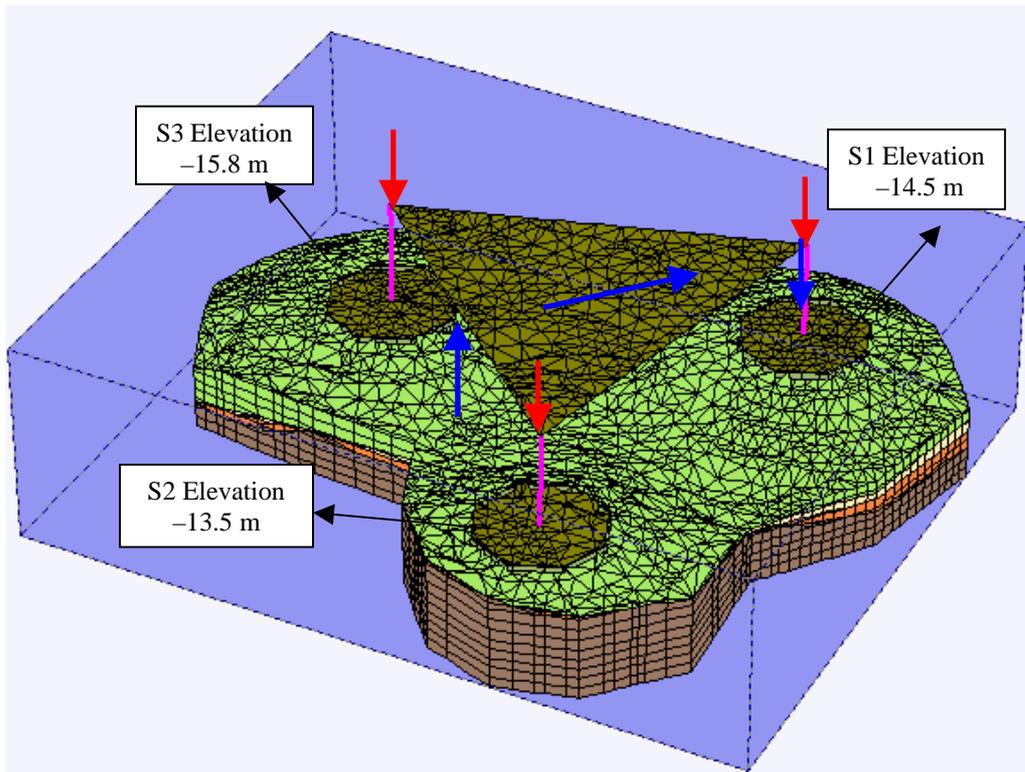


Fig. 8. 3D FE structure-foundation model, final sand banks

The scenario with sand bank elevations at -14.5 m, (height about 8.5 m), -13.5 m, (height about 5.5 m), -15.8 m, (height about 10.7 m) for S1, S2 and S3 locations, respectively was finally proposed.

The skirted spudcans are simplified by octahedrons. The spudcan is flat and in full contact with the sand bank from the start of the preloading. The 3 chords are not included. The tip of the outer skirts is from the start of the analyses at elevation calculated from bank elevation minus 2.3 m (the skirt length).

The jack-up structure is modelled as shown in Fig. 8. The leg elements can change length due to applied axial force. The leg beams and the spudcan plates at the connection points can simulate the 6 degrees of freedoms.

The soil conditions, (soil profiles derived from the seismic, PCPT / borehole data at different cross sections), are modelled by implementing boreholes, as seen from the horizontal planes in Fig. 7. Some of the cross sections showing the soil profiles / sections with final gravel banks are given in Fig. 9.

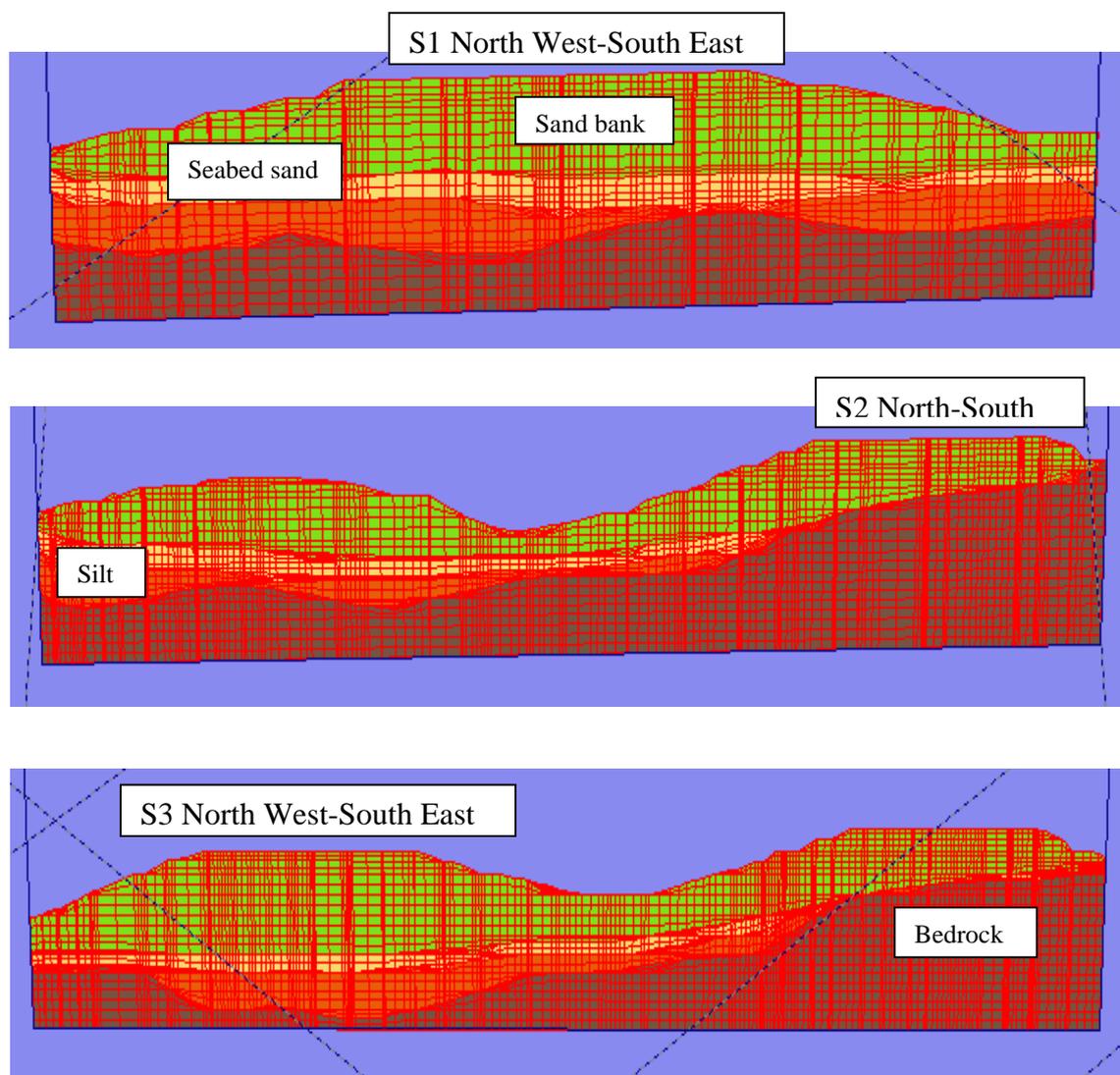


Fig. 9. Cross section profile through the model and the skirted spudcan locations

The calculation procedure consists of 3 load stages, which are: Preloading to max vertical load $V = 145 \text{ MN / leg}$; Unload to vertical $V = 112 \text{ MN / leg}$, $V = 100 \text{ MN / leg}$, $V = 115 \text{ MN / leg}$ for S1, S2, S3, respectively; Storm loading, horizontal $H = 6.4 \text{ MN}$, moment $M = 345.6 \text{ MNm}$ at the most critical plane for wind speed 33 m/s . The storm load comes from any direction so different analyses were needed to define the critical one.

The loads for preloading are applied as vertical forces at each leg. The horizontal force is applied at the hull plate pointing towards the critical leg. The moment is implemented as a set of two vertical loads, applied downward at the critical leg-hull connection and upwards at a point in the hull between the other two legs as shown in Fig. 8. An initial phase is calculated consisting of the construction of the sand banks.

The limited combined loads at the structure, one single leg, are calculated from the rig designer as: Horizontal shear force $Q = 18 \text{ MN / leg}$; Bending moment at hull $M = 325 \text{ MNm / leg}$;

The soil strength for the silt layer $c_u = 25 \text{ kPa}$, initially applied based on the engineering judgement was not considered a lower bound assessment based on the PCPT data and usual North Sea ($q_{\text{net}} - c_u$) correlation. After reviewing the available soil data, to increase to some level safety concerning the soil parameters following DNV⁶ it was decided to reduce the shear strength for the silt layer from $c_u = 25 \text{ kPa}$ to $c_u = 15 \text{ kPa}$. This strength is considered a lower bound design value, when taking into account the consolidation during construction of the banks.

The FE analyses are carried out with Plaxis programs^{7,8}. The sand bank material was specified to correspond to the soil strength applied in the analyses. The total volume of the sand material was about 60000 m^3 .

Results from the 3D FE structure – foundation model

The results for the initial phase, including the construction of the sand banks indicated vertical non-uniform settlements of about 30 cm , taken into account in the calculation of the total sand volume.

The structural reaction forces for different models and preloading phase are given in Table 2.

Table 2. Results from 3D FE model, different gravel bank elevation scenarios

3D models Sand Bank Elevation (m) S1 S2 S3	Silt Undrained Shear Strength c_u (kPa)	Structure Reaction, Bending Moment M (MNm)	Structure Reaction, Shear Force Q (MN)
-20.0, -13.5, -24.0	$c_u = 25$	Over Limit	Over Limit
-16.8, -13.5, -19.1	$c_u = 25$	295.0	17.7
-15.8, -13.5, -18.1	$c_u = 25$	224.7	14.4
-15.8, -13.5, -18.1	$c_u = 20$	Over Limit	Over Limit
-14.5, -13.5, -16.8	$c_u = 15$	Over Limit	Over Limit
-14.0, -13.5, -16.3	$c_u = 15$	229.8	16.1
-14.5, -13.5, -15.8	$c_u = 15$	211.0	15.1

The structure and spudcan vertical and maximum horizontal displacements are given in Fig. 10. The maximum calculated reaction forces are $M = 211.04 \text{ MNm}$, shear force Q_{max}

= 15.13 MN, differential penetration at S1 about 20 cm, sliding of S1 about 12 cm. The calculations for the unloading and storm loading analyses show only slight changes in the deformations and structure reaction forces.

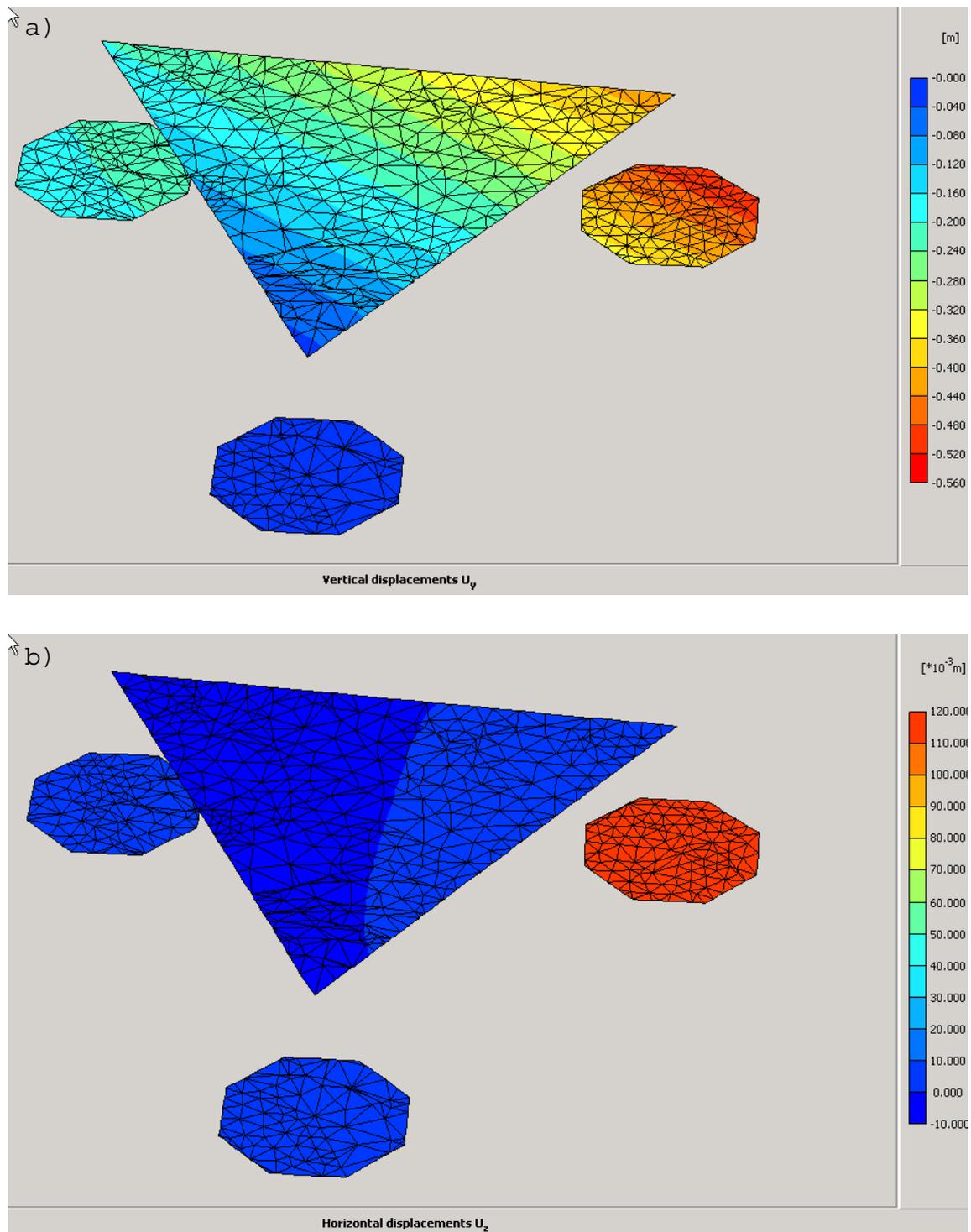


Fig. 10. Preloading phase, structure displacements (only plate elements shown):

a) Vertical displacements b) Horizontal displacement

The mechanism of non-uniform penetration or rotation for S1 and S3 can be explained with the varying thickness of the total sand material and the underlying silt layer, within the spudcan area. The outward rotation of S1 is for example expected due to thinner friction material in the north-northwest.

The final bank elevations seem to be the optimum ones. Reducing the bank heights at one location, for example at S3 if elevation is reduced at -16.8 m, this has a strong effect in the structure reaction forces as seen from the results in Table 2.

No punch through or rapid penetration was predicted and the spudcans will penetrate to about full base contact. Some differential spudcan penetrations, rotation and horizontal movement could though occur during preloading, believed not to be critical for the jack-up structure. However, from the load displacement curves the inward horizontal movement of S1 is in the plastic phase and could further develop. Failure figure at for example S1 is given in Fig. 11. Stability analysis showed safety factor slightly over 1.0.

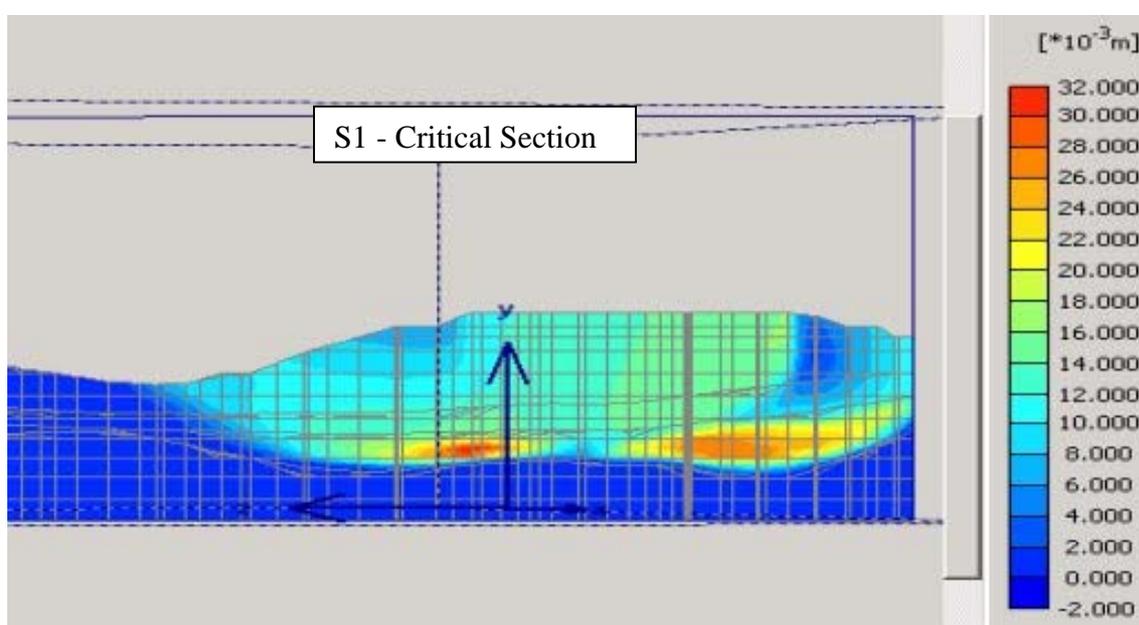


Fig. 11. Critical failure figure at S1, total displacement increments

CONCLUSIONS

3D FE structure - foundation interaction analyses are carried out for the installation of the world's largest jack-up rig, offshore, Norway, where seabed features like slopes and depressions and non-uniform soil conditions were the concern.

Banks of sands were designed at the skirted spudcan locations with different heights, ensuring that the structure reaction forces developed during preloading, causing footings rotation / sliding, did not exceed the calculated limits.

The jack-up rig was successfully installed at the location and spudcan penetrations / displacements similar to the predicted values were recorded.

ACKNOWLEDGEMENT

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